Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

# Dynamic and seismic tests on a prototype of a multistorey building

L. Sanpaolesi & M. L. Beconcini
*Department of Theory of Structures, University of Pisa, Italy*
M. Casirati, M. Fumagalli & S. Viani
*Structural Testing and Survey Department, ISMES, Bergamo, Italy*

ABSTRACT: The paper describes the dynamic and seismic tests carried out on a three floors prototype building, 7,65 m x 9,40 m in plan, without infilling walls. Aim of the tests was the direct determination of dynamic and seismic characteristics of the building. This allows the uncertainties of the computations related to the non-linear behaviour of the structural elements and joints, soil structure interaction, actual seismic response, to be overcome. Tests have been carried out by exciting the building by means of a mechanical vibrator fastened to the roof and picking up its overall and local structural response. Dynamic characterization tests have been performed with exciting forces acting - non simultaneously - in both x and y horizontal directions, and the first 9 vibration modes have been described. Seismic tests with acting force in direction of the main beams have been also carried out at increasing vibration levels,until some cracks were induced in the building.

## 1. INTRODUCTION

Forced vibration tests are of basic importance for the evaluation of the seismic behaviour of a building, especially when it is a r.c. prefabricated one.

As is known, direct testing allows the uncertainties present in calculations to be overcome, as far as the schematization of the non-linear behaviour of structural elements and their connections, soil-structure interaction, actual seismic response and contribution of non-structural parts are concerned.

Some of these aspects have been considered in these tests, which on purpose did take into account only the bearing frame of the building. The contribution of non-structural elements will be taken into account in a second set of tests on a similar building provided with infilling walls of different types.

## 2. THE TESTED PROTOTYPE

The structure under test is a full-scale r.c. building, prefabricated according to the so called "sistema K" design, which has been developed for bearing frame structures, whose main features are sketched in Fig.1.

Main elements of this system are the column and the beam, which can be used with different types of floors.

The columns,whose height is of a single storey, are completely precast and are provided with "slots" for the connection of the reinforcement bars coming from the upper column. The provisional connections, as well as the positioning of the column before grouting are supplied by a steel profile.

The beams, which are only partly prefabricated, are of different types according to their position in the floor.

The floor is generally made up of panels of hollow bricks and concrete, or by "predalles".

The connection of the different elements is obtained by the grouting of the total floor area at the same time; this constitutes an important and positive feature of this prefabrication technique.

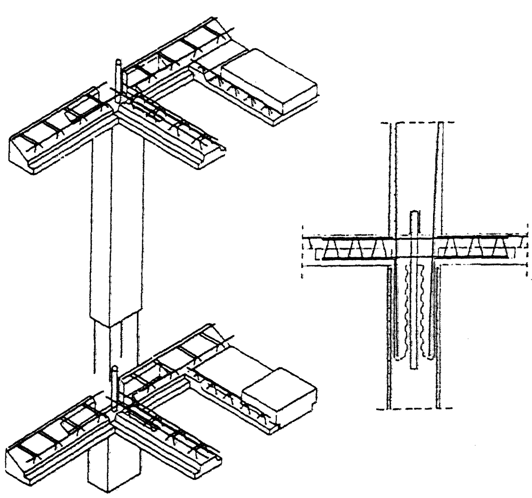


Fig.1: Sketch of the assembling of the "Sistema K" structural elements

Fig.2 shows the overall sketch of the tested structure, which has been built on the testing ground of the Structural Testing and Survey Dept. of ISMES in Bergamo (Italy).

The tested structure is the bearing frame of a three-storey building, 7,65 x 9,40m in plan, 9,06m high. Total dead load and living loads (these latters equal to 1/3 of the maximum code value) are simulated by gravel.

Large foundations (of the grade beam type) have been cast in situ, to avoid damage in this area and thus restrict the observations to the behaviour of the over-ground structure.

The floors are made up by hollow brick and concrete slabs at the two upper floors, and by predalles at the first floor.

Strength of the cast in situ concrete is 30 MPa, whereas that of the prefabricated elements is 45 MPa.

## 3. DYNAMIC CHARACTERIZATION TESTS

Dynamic characterization tests have been performed by acting the structure through two sinusoidal (non simultaneous) forces F1, F2 applied to the upper floor along the main directions (Fig.3).

The forces were generated by a mechanical unbalanced masses vibrator manufactured by ISMES, able to deliver a maximum force of 200 kN. The frequency range scanned is from 1 to 16 Hz.

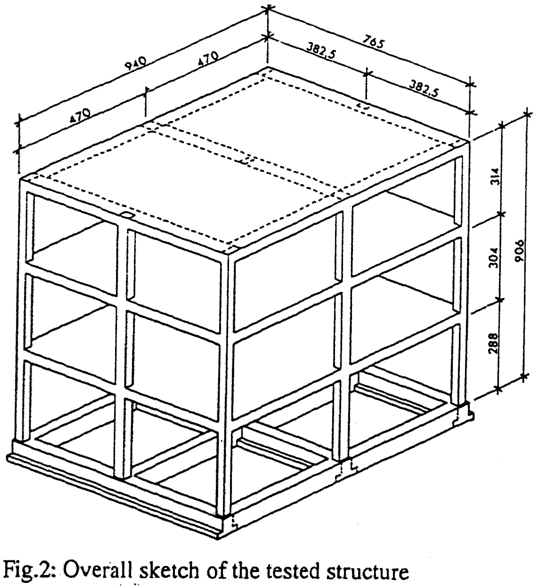



Fig.2: Overall sketch of the tested structure

The structural response has been picked up by a set of transducers as follows:

- horizontal floor accelerations (accelerometers A1-A19)
- foundation vibration velocity (seismometers V1-V9)

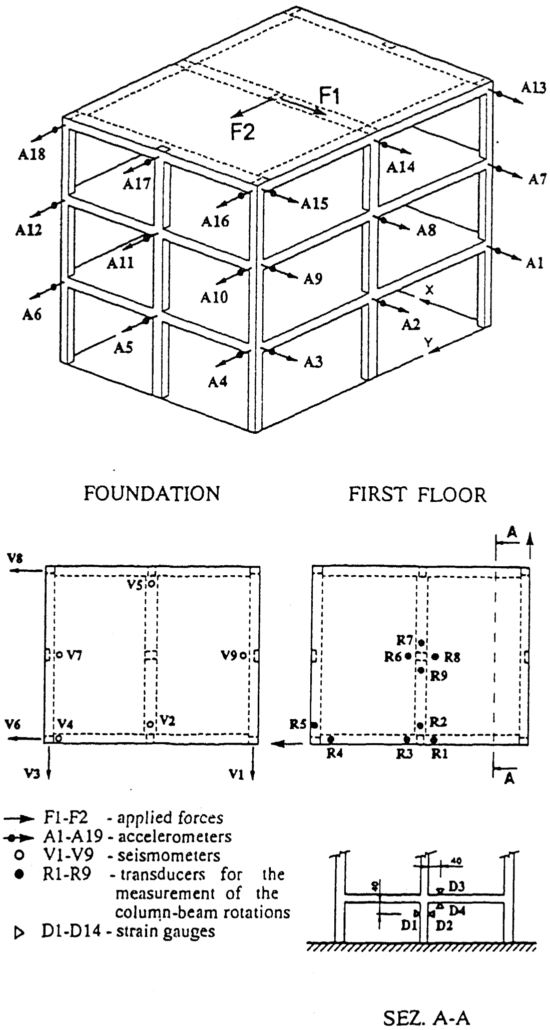



Fig.3: Excitation and measuring positions

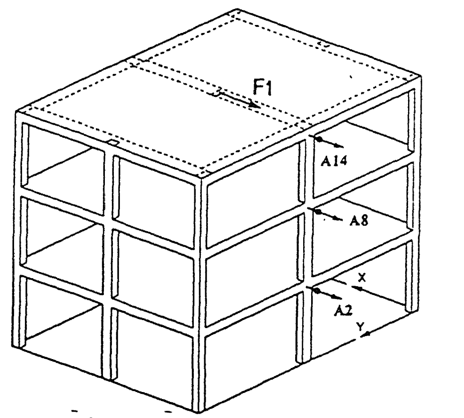
- relative rotations of some columns and first floor beams (transducers R1-R9)
- concrete strains of the central column and central beam at first floor (strain gages D1-D4)

Test control and data acquisition were supplied by a computerized system which makes the frequency to vary at a low rate, and simultaneously measures the response.

Thus, the transfer functions of the system

$$h_{j,r}(f) = \frac{P_j}{F_r} \cdot e^{i \cdot \vartheta_j} \qquad (1)$$

are obtained, where:



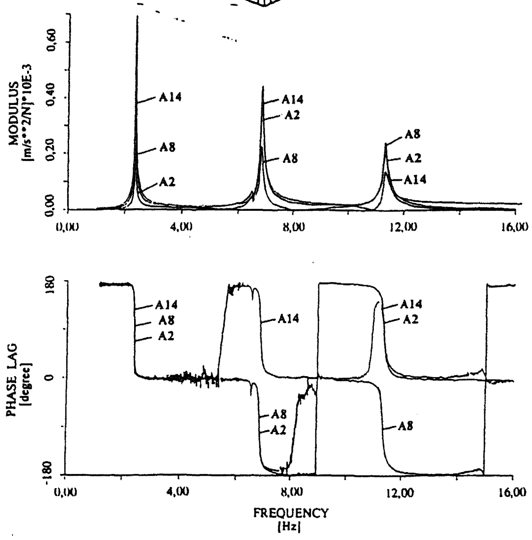




Fig.4: Example of transfer functions

$P_j / F_r$ is the ratio between the amplitudes of the j-th response and the r-th applied force

$\vartheta_j$ is the phase lag between the above mentioned signals being the imaginary unit.

Fig.4 shows, as an example, the transfer functions of the building in one direction when it is excited in that direction.

It can be noticed that the moduli of the transfer functions show three sharp amplifications at the frequencies of 2,36 , 6,84 and 11,30 Hz, which correspond to the three vibration modes of the building in the excitation direction, whereas the phase lag plots show in phase and counter-phase movements of the floors.

The processing of experimental transfer functions lead to the determination of 9 vibration modes (3 translation modes in each of the horizontal main directions, 3 torsional modes along the vertical axis). The modal shapes are shown in Fig.5.

It has also been determined that, in the frequency

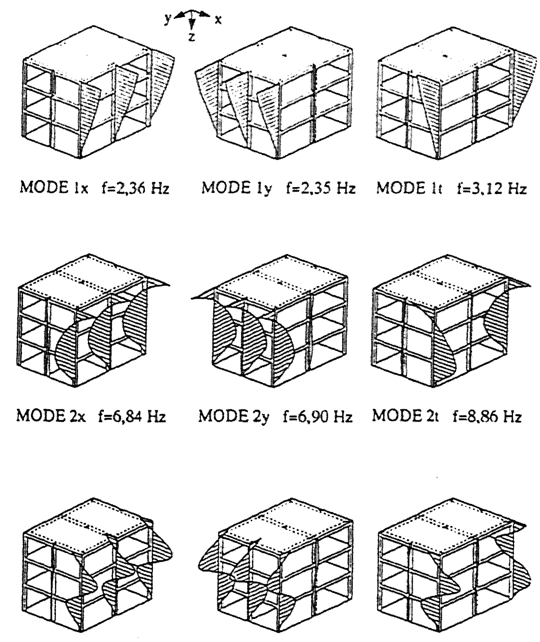



Fig.5: Modal shapes and resonance frequencies of the building.

range of the tests, the floors behave as rigid slabs in their planes.

Table 1 summarizes the results of the analysis of the flexural modes of the building. For each mode are given:

- the resonance frequency f
- the damping coefficient $\zeta$
- the seismic participation coefficients $C_x$ and $C_y$
- the equivalent mass M
- the equivalent stiffness K

The same table 1 lists the values of the orthogonality coefficients: the experimental values for modes $m \neq n$ indicate both a good discretization of the structure and a very satisfactory determination of the modal shapes.

## 4. RESPONSE TO EARTHQUAKE

The knowledge of the experimental modal parameters (natural frequencies,modal shapes and related damping coefficients) allows the seismic response of the building to be determined.

This has been obtained through the usual and simple description of the seismic input by the response spectrum technique.

The seismic input is characterized by the acceleration spectrum $S_a(f,\zeta)$, whilst the maximun response of the k-th mode is computed through the relationship:

$$a_j^k(max) = \max a_j^k(t) = C^k \cdot \varphi_j^k \cdot S_a(f^k,\zeta^k) \qquad (2)$$

Table 1: Modal parameters obtained from dynamic characterization tests

| MODE | FREQUENCY "f" [Hz] | DAMPING COEFF. "ξ" % | PARTECIPATION COEFF. "$C_x$" | "$C_y$" | EQUIVALENT MASS "M" [Kg] | EQUIVALENT STIFFNESS "K" [$10^6$ N/m] | MODE ORTHOGONALITY mode 1 | mode 2 | mode 3 |
|---|---|---|---|---|---|---|---|---|---|
| 1x | 2,36 | 0,99 | 1,4 | - | 69300 | 15,22 | 1 | -0,06 | -0,04 |
| 2x | 6,84 | 0,98 | 0,51 | - | 106900 | 197,25 | -0,06 | 1 | -0,02 |
| 3x | 11,3 | 0,94 | 0,15 | - | 100050 | 503,84 | -0,04 | -0,02 | 1 |
| 1y | 2,35 | 0,93 | - | 1,27 | 71700 | 15,62 | 1 | -0,08 | 0,03 |
| 2y | 6,9 | 0,86 | - | 0,39 | 114650 | 215,27 | -0,08 | 1 | 0,16 |
| 3y | 11,72 | 0,96 | - | 0,13 | 110450 | 598,33 | 0,03 | 0,16 | 1 |

where $C^k$ is the seismic participation coefficient of the mode and direction considered.

The maximum responses which take into account all the modes are obtained according to statistical criteria, as - for example - the following one:

$$q_j(max) = \left[ \sum_{k=1}^{n} q_j^k(max)^2 \right]^{1/2} \qquad (3)$$

For the structure under test the total maximum response at the base practically coincides with that of the 1st mode only, as shown in the following table, in which are listed the base shear forces and bending moments computed for the 1st mode and for the first 3 modes, according to (2) e (3) respectively, starting from the spectrum of the Italian code $S_a$ = 0.98 $m/s^2$ for the seismic areas of 1st category.

| | | Direction x | y |
|---|---|---|---|
| Shear force (kN) | Mode n.1 | 140.8 | 125.3 |
| at the base of | Σ Modes 1-3 | 143.7 | 127.4 |
| the building | % Difference | 2.1 | 1.7 |
| Bending moment | Mode n.1 | 909.6 | 815.7 |
| (kN·m) at the base | Σ Modes 1-3 | 909.8 | 816.0 |
| of the building | % Difference | 0.02 | 0.04 |

## 5. SESMIC CHECK

The final tests to check the seismic resistance of the building have been carried out using the same unbalanced masses mechanical vibrator used for the characterization tests.

The applied force acted along the direction of the main beams (x direction), at the frequency of the 1st resonance, since, as shown above, this is the only of importance in the overall response of the building.

In detail, the first step of the tests was the generation - in correspondence of the 1st mode frequency - of accelerations according to the Italian seismic code ($S_a$ = 0,69 $m/s^2$ for 2nd category areas; $S_a$ = 0,98 $m/s^2$ for 1st category areas). In the second step higher accelerations, corresponding to more severe spectra, were applied.

The obtained acceleration spectra $S_a$ have been evaluated according to the relationship

$$S_a = \frac{a_j^1(max)}{C^1 \cdot \varphi_j^1} \qquad (4)$$

where:

$a_j^1(max)$ is the first mode maximum acceleration generated in the j-th point

$C^1$ is the corresponding participation coefficient

$\phi_j^1$ is the 1st modal shape in the j-th point

The maximum vibration levels generated during the tests - which have been carried out with steady - state sinusoidal force at the 1st resonance frequency, are given by

$$a_j^1(max) = \frac{F_r \cdot \varphi_r^1 \cdot \varphi_j^1}{2 \cdot \zeta^1 \cdot M^1} \qquad (5)$$

where:

$F_r = K \cdot f^2$ is the force generated by the vibrator

$\varphi_r^1$ is the 1st modal shape in the r-th point

$\varphi_j^1$ is the 1st modal shape in the j-th point

$\zeta^1$ is the 1st modal damping coefficient

$M^1$ is the 1st mode equivalent mass.

As shown in Table 2, the quantities involved in (5) vary when the excitation levels change, due mainly to non linear effects.

Owing to this fact, it has not been possibile to determine a priori, through the relationships (4) and (5), the intensity and the frequency of the force that the vibrator should generate to obtain on the building the required response spectrum. The required values were thus obtained experimentally, after carrying out some tests at increasing force levels.

Fig. 6 shows the acceleration spectra reached during

Table 2: Modal parameters for mode 1x from seismic check tests

| TEST | FREQ. "f" [Hz] | FORCE "Fx" [N] | DAMPING COEFF. "ξ" % | PART. COEFF. "Cx" | EQUIVALENT MASS "M" [Kg] | EQUIVALENT STIFFNESS "K" [$10^6$ N/m] | MAXIMUM ACCELERATION "$a_{max}$"[$mm/s^2$] | ACCELERATION SPECTRUM "$S_a$" [$m/s^2$] |
|---|---|---|---|---|---|---|---|---|
| A | 2,33 | 464 | 0,85 | 1,39 | 70450 | 15,08 | 377 | 0,27 |
| B | 2,31 | 915 | 1,1 | 1,38 | 71650 | 15,08 | 565 | 0,41 |
| C | 2,25 | 1738 | 1,48 | 1,38 | 71580 | 14,29 | 800 | 0,58 |
| D | 2,18 | 3259 | 1,75 | 1,24 | 79390 | 14,88 | 1133 | 0,91 |
| E | 2,05 | 5749 | 2,13 | 1,27 | 76290 | 12,64 | 1662 | 1,31 |
| F | 1,86 | 13830 | 3,03 | 1,31 | 72910 | 9,95 | 2873 | 2,19 |
| G | 1,77 | 4286 | 2,4 | 1,37 | 67570 | 8,35 | 1157 | 0,84 |

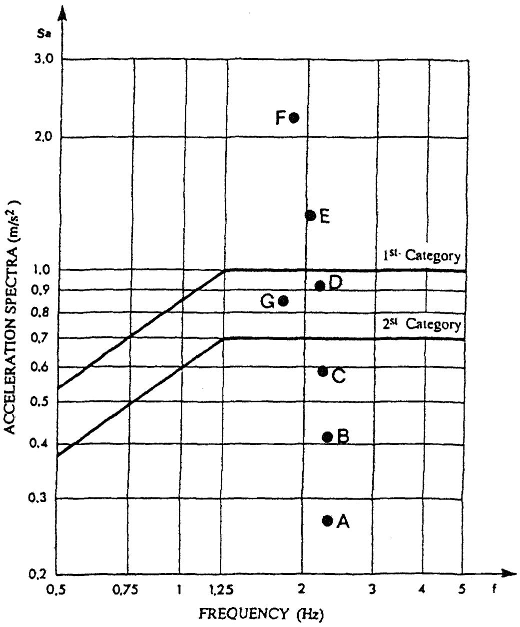



Fig.6: Acceleration levels reached during the seismic check tests

the seismic check tests, computed according to (4). The values obtained in the different tests (from A to G) are compared with those of the Italian code: during the tests at higher acceleration levels, acceleration spectra much higher than those required by the Italian code have been reached.

During test "F", a sudden decrease of the structural response occurred, followed by beats (Fig.7), and the rotations transducers at the column-foundation joints showed a remarkable increase of the rotation and a marked unsymmetry.

Both effects (general and local) usually occur when cracks appear in some structural members: cracks were actually observed at the column-foundation joints when the structure vibrates, which cannot be seen when the movement stops.

A furter test ( G ) carried out at lower acceleration levels, showed that these cracks induced permanent alterations of the behaviour of the structure, whose frequency dropped from 2,3 hz to 1,8 hz.

For a correct interpretation of these results, it has to be underlined that the structure has been subject to a large number of high level loading cycles, owing both to the search for the correct values of the forces to be applied, and to the testing technique which requires a quasi-steady application of the dynamic force.

Therefore, the effects of the tests on the structure may be considered as more severe than those of a real earthquake having the same acceleration spectrum.

## 6. CONCLUSIONS

The experimental analysis carried out on the prototype building allowed the dynamic behaviour to be determined through the detection of the parameters of 9 vibration modes, and its seismic resistance to be checked, by the application of high dynamic force levels, which induced in the building larger accelerations than those of the Italian seismic code.

The weakest structural point resulted to be the connection between the columns and the foundation.

Further tests are planned on a similar prototype with infilling walls of different types.

Their results - compared with those here described - will allow the influence of the non structural elements on the overall dynamic behaviour to be analyzed.

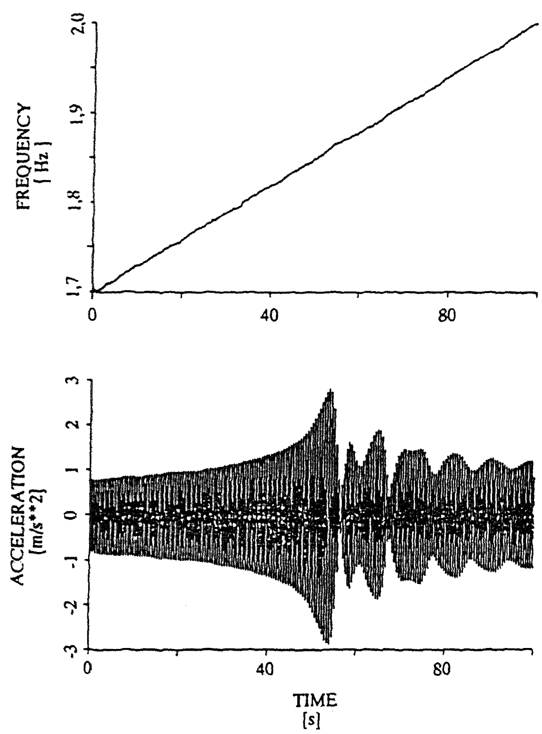



Fig.7: Excitation frequencies and time history of accelerations reached during the seismic check test F

## REFERENCES


Angotti F., Castoldi A., Pezzoli P., Sanpaolesi L.: "Analisi sperimentale della risposta dinamica e del comportamento sismico di un edificio prefabbricato a grandi pannelli", 'L'industria Italiana del Cemento', 7-8/1982.

Benedetti A., Finzi R., Fornaciari M., Malavolta A., Pezzoli P., Viani S.: "Prova dinamica di simulazione del sisma su di una struttura puntiforme con pilastri pluripiano", 'L'Industria Italiana del Cemento', n.3, 1984.

Castoldi A. : Analisi sperimentale delle strutture ", chapt.X of the book 'Costruzioni in zona sismica', co-ordinative A.Castellani, Masson Italia Editori, Milano, 1983.

Castoldi A., Casirati M., Pezzoli P.: "Experimental Seismic Verification of Assembled Prefabricated Structures", Proceedings, 7th World Conference on Earthquake Engineering, Istanbul, September 1980, Vol. 7, 149-156.

Castoldi A., Casirati M., Pezzoli P.: "Experimental seismic verification of assembled prefabricated structures", 'I Quaderni dell'ISMES' n.164, 1982.

Decreto Ministeriale Italiano 19 June 1984 "Norme tecniche relative alle costruzioni in zone sismiche" and following modifications (D.M. 24 January 1986).

Hurty W.C., Rubinstein M.F. 'Dynamic of Structures', Prentice Hall, Ing.1964.

Migliacci A., Pezzoli P., "Un esempio di sperimentazione su prototipi", Congress CTE, Perugia, November 1985

Panzeri P., Pezzoli P. : "Valutazione sperimentale della risposta sismica di un edificio prefabbricato mediante prove di vibrazione forzata" 'Ingegneria sismica' year III, n.2 - May-August 1986.

Richardson M., Potter R.: "Identification of the modal properties of an elastic structure from measured transfer function data", Albuquerque-New Mexico 1974, XX International Instrumentation Symposium.

Sanpaolesi L., Caramelli S.: "Esperienze su prototipi di nodi trave pilastro sotto azioni sismiche simulate", Congress CTE,1986.

Sanpaolesi L., Beconcini M.L., Fumagalli M., Viani S. "Indagini sperimentali in campo sismico su prototipo di edifico prefabbricato pluripiano" 5º Italian Congress 'L'ingegneria sismica in Italia', Palermo 29.9 - 2.10 1991

Timoshenko S., 'Vibrations Problems of Engineering', D.Van Nostrand Company, Inc. 1966.